

CHAPTER 6 UNCONFINED FLOW PROBLEMS

6-1. Introduction. This chapter will consider unconfined flow problems for cases involving earth dams. Because of their ability to give a strong visual sense of flow and pressure distribution, flow nets will be used to define seepage. Other methods, such as transformations (Harr 1962), electrical analogy models, and numerical methods, can provide pressures and flows and be used to develop flow nets. Unconfined flow problems require the solution of flow and pressure distribution within the porous media and definition of the line of seepage boundary (phreatic surface within the dam).

6-2. Homogeneous Earth Dam on Impervious Foundation. The simplest earth dam configuration consists of a homogeneous, pervious embankment on an impervious foundation. Though rarely encountered in engineered embankments, this case will introduce general methods of defining flow in embankments.

a. Definition of Unknown Seepage Boundaries and Calculation of Flow per Unit Length of Embankment, q . It is desired to define the flow and pressure distribution within the embankment and total flow through the embankment. The first step is determination of the upper flow line (which is the line of seepage boundary) and the length of the seepage exit face on the downstream slope of the earth dam. This provides all necessary boundary conditions for flow net construction and complete seepage definition. The two unknown boundaries, BC and CD, figure 6-1, are a combination of an entrance condition, figure 4-7(c), BB_1 ; part of a parabola, B_1B_2 ; a smooth transition between points of tangency, B_2C , and a straight line discharge face along the downstream slope, CD. A parabola, shown by the dashed line, is the basic geometric member used to define the location and extent of the two boundaries. Casagrande (1937) provided the standard reference for flow through embankments while others (Harr 1962, Cedergren 1977, and others) added to and refined the basic methods. Figure 6-2 provides the nomenclature and formulas for drawing the line of seepage and exit face and determining the quantity of seepage per unit length of embankment, q . In a given problem, embankment geometry and head water elevation provide values for h , m and α which allow location of points A and B and determination of distance, d , as shown in figure 6-2.

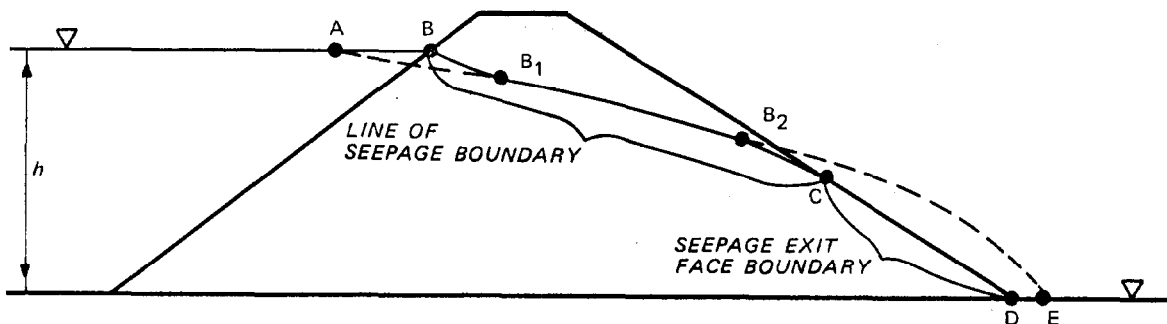
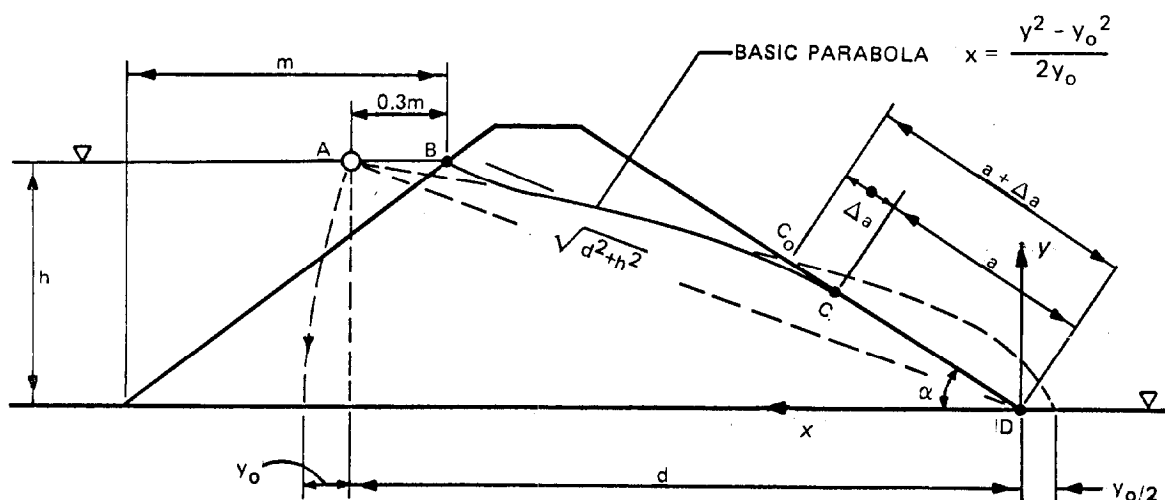


Figure 6-1. Line of seepage, BC, and seepage exit face, CD, for a homogeneous earth dam on an impermeable foundation (prepared by WES)



α	METHOD	EQUATIONS
$< 30^\circ$	SCHAFFERNAK - VAN ITERSON	$a = \frac{d}{\cos \alpha} - \sqrt{\frac{d^2}{\cos^2 \alpha} - \frac{h^2}{\sin^2 \alpha}}$ $q = k a \sin \alpha \tan \alpha$
$\leq 90^\circ$	L. CASAGRANDE	$a = S_0 - \sqrt{S_0^2 - \frac{h^2}{\sin^2 \alpha}}$ <p>FOR $\alpha \leq 60^\circ$, USE $S_0 = \sqrt{d^2 + h^2}$. FOR $60^\circ < \alpha < 90^\circ$, USE MEASURED $S_0 = \widehat{AC} + \widehat{CD}$</p> $q = k a \sin^2 \alpha$
180°	KOZENY	$a_0 = \frac{y_0}{2} = \frac{1}{2} (\sqrt{d^2 + h^2} - d)$ $q = 2ka_0 = ky_0$
$30^\circ \text{ TO } 180^\circ -$	A. CASAGRANDE	<p>DETERMINE $(a + \Delta a)$ AS THE INTERSECTION OF THE BASIC PARABOLA AND DAM SLOPE. THEN DETERMINE Δa FROM C VALUE ON FIG. 6-5.</p> $q = k a \sin^2 \alpha$ <p>OR</p> $q = ky_0 = k (\sqrt{d^2 + h^2} - d)$

Figure 6-2. Determination of line of seepage and seepage exit face for embankments on impervious foundations (adapted from New England Waterworks Association¹⁵¹)

b. After this is done one of the four methods shown in figure 6-2 and explained below can be used to determine the location of the exit face CD and the line of seepage BC.

(1) $\alpha < 30^\circ$ Schaffernak-Van Iterson. The two formulas for this method given in figure 6-2 assume gradient equals dy/dx and allow direct determination of a and q . Construction of basic parabola shown in figure 6-3 is the first step in determining the upper line of seepage (Casagrande 1937). From embankment geometry and headwater height, point A is located. d and y are determined by scribing an arc, with radius DA through point E. Then the point of vertical tangency of the basic parabola, F, is determined. Line AG, parallel to the embankment base and horizontal axis of the parabola, is drawn and divided into an equal number of segments (6 in the case in figure 6-3). Line GF, the vertical tangent to the parabola, located at $y_0/2$ from the downstream toe of the embankment is divided into the same number of equal segments as line AG. The points dividing line AG into segments are connected with point F. The intersection of these lines with their counterpart lines drawn from the points on line GF define the parabola. Thus the basic parabola, dashed line A-F, is defined. The upstream portion of the line of seepage, dotted line BH, is drawn by starting at point B perpendicular to the upstream slope (since the upstream slope is an equipotential line and the line of seepage is a flow line) and continuing downstream to make line BH tangent to the basic parabola at point H which is selected based on judgment. This is an entrance condition as shown in figure 4-7. The central portion of the line of seepage is along the basic parabola while the downstream portion is a smooth transition from the basic parabola to tangency with the downstream slope at point C. Point C is located a distance a from the downstream toe as determined by the equation for Schaffernak-Van Iterson shown in figure 6-2. With all seepage boundaries known and using the rules of Chapter 4, a flow net may be constructed within the boundaries as shown in figure 6-4. This figure points out the important basic flow net requirement that all equipotential lines intercept the line of seepage and exit face at points with equal vertical separation (in this case $H/10$ apart).

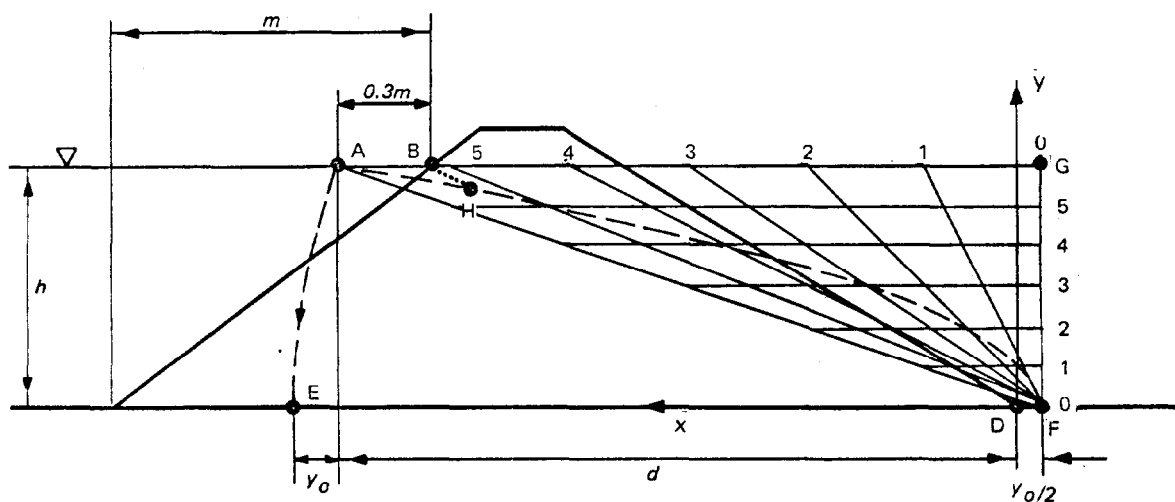


Figure 6-3. Construction of basic parabola (prepared by WES)

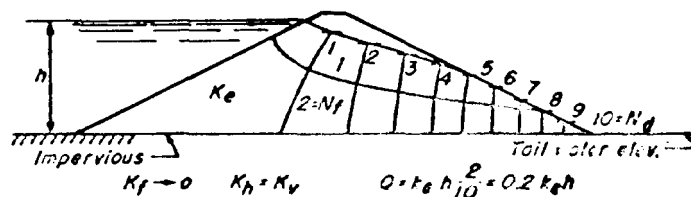


Figure 6-4. Seepage through a permeable embankment underlain by an impermeable foundation (prepared by WES)

(2) $\alpha \leq 90^\circ$ L. Casagrande. The gradient assumption for this method is $i = dy/ds$ where s is the distance along the line of seepage, and allows greater accuracy than Schaffernak-Van Iterson method for steeper downstream slopes. Use of the equations in figure 6-2 and the same general procedures used for the Schaffernak-Van Iterson method apply for α 's up to 60° . For $60^\circ < \alpha \leq 90^\circ$, since a and s are interdependent, the location of point C (or distance a) must be estimated to determine the value of $s_0 = \overline{AC} + \overline{CD}$, then distance a calculated. This procedure is repeated until there is satisfactory agreement between the \overline{CD} portion of the distance s_0 as measured and a as calculated. Thus the seepage boundaries are established allowing flow net construction.

(3) $\alpha = 180^\circ$ Kozeny. For this special case Kozeny described a solution adapted by Casagrande (1937). Figure 6-5 illustrates the nomenclature and construction method for this case. Embankment geometry, h , and drain location control construction of the basic parabola. For this case the seepage face is the distance a_0 and the correction Δa is not used. Again with boundary definition, the flow net can be drawn.

(4) $30^\circ \leq \alpha \leq 180^\circ$ A. Casagrande. After study of model experiments and construction of flow nets for various α 's, A. Casagrande (1937) developed a curve, figure 6-6, which relates a to the ratio, $c = \frac{\Delta a}{a + \Delta a}$. Construction of the basic parabola is the first step in this procedure. The point, C_0 , as shown in figure 6-2, where the basic parabola intercepts the downstream slope is determined and distance $a + \Delta a$ is measured. Knowing α , C can be found in figure 6-6 and Δa calculated. Information is then sufficient to draw the line of seepage and discharge face, determine q , and construct the flow net. Casagrande (1937) provides a procedure for the condition of tailwater on the downstream slope:

For the comparatively rare case in which the presence of tailwater must be considered in the design, the determination of the line of seepage and of the quantity can be performed by dividing the dam horizontally at tailwater level into an upper and lower section. The line of seepage is determined for the upper section in the same

30 Sep 86

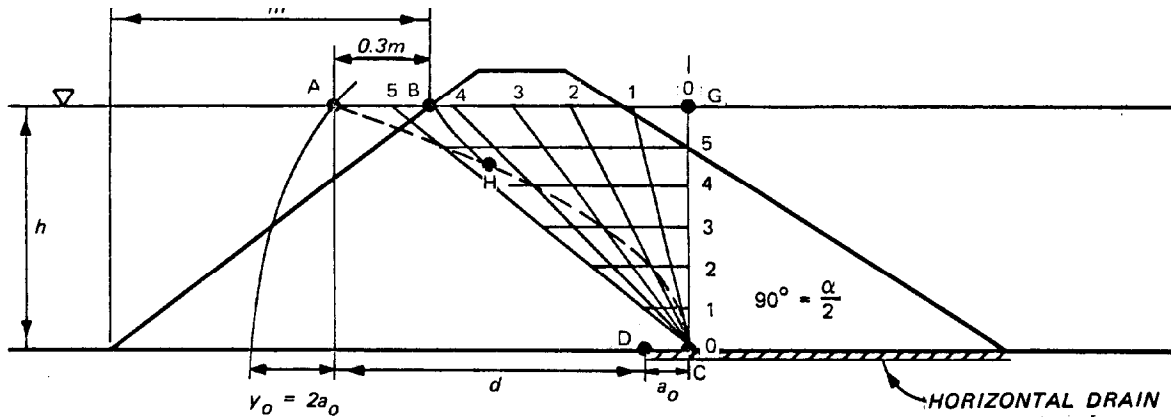
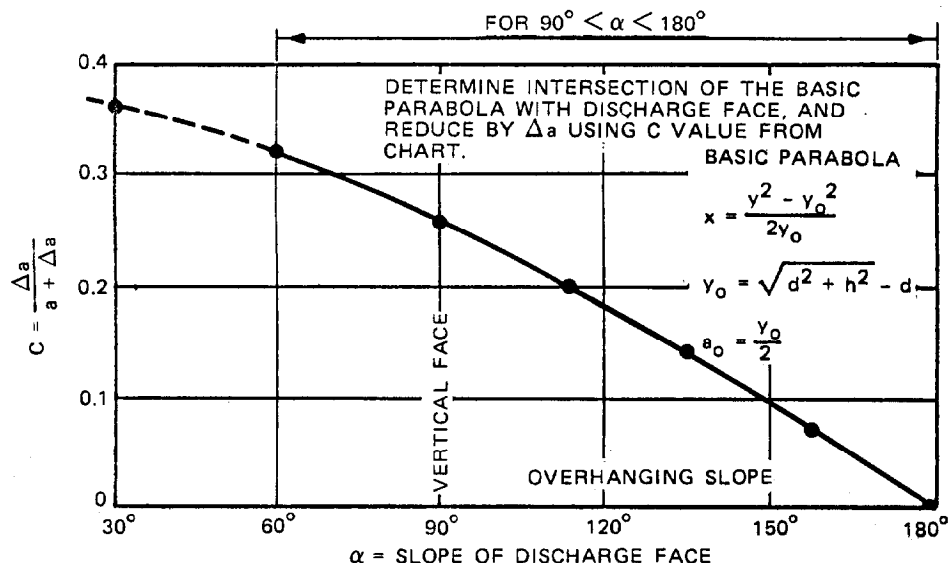


Figure 6-5. Construction of basic parabola and seepage line for $\alpha = 180^\circ$ Kozeny (courtesy of New England Waterways Association¹⁵¹)



NOTE: POINTS WERE FOUND BY GRAPHICAL DETERMINATION OF FLOW NET.

Figure 6-6. c vs α (courtesy of New England Waterworks Association¹⁵¹)

30 Sep 86

manner as if the dividing line were an impervious boundary. The seepage through the lower section is determined by means of Darcy's law, using the ratio of the difference in head over the average length of path of percolation as the hydraulic gradient. The total quantity of seepage is the sum of the quantities flowing through the upper section and the lower section. The results obtained by this rather crude approximation agree remarkably well with the values obtained from an accurate graphical solution.

Harr (1962) explains an additional method, known as Pavlovsky's solution, for determining a_0 and q for the case of a homogeneous, pervious embankment on an impervious foundation. Pavlovsky analyzed the embankment by dividing it into three zones, writing an equation for q in each of the zones and, by assuming continuity of flow, equating the three equations for q . Figure 6-7 provides the nomenclature for Pavlovsky's solution. The embankment is divided as shown with Zone I between the upstream slope and a vertical line at

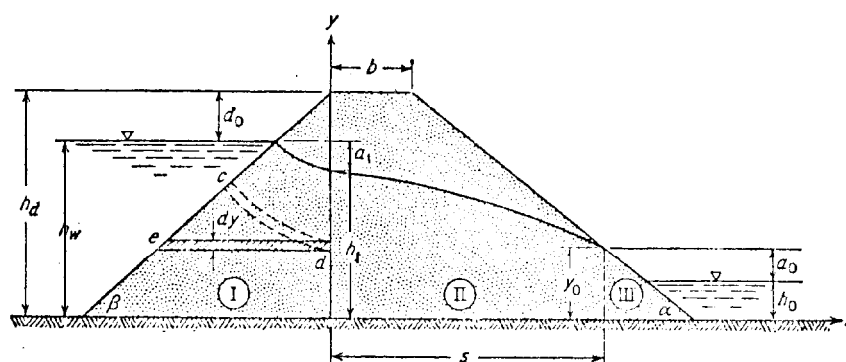


Figure 6-7. Nomenclature for Pavlovsky's solution
(courtesy of McGraw-Hill Book Company¹⁸⁰)

the intersection of the crest and upstream slope (y axis), Zone II between the y axis and a vertical line at the intersection of the line of seepage with the downstream slope, and Zone III which is composed of the remainder of the downstream toe. Pavlovsky assumed horizontal flow in each zone and wrote the basic equation $q = kIA$ for each zone using the nomenclature of figure 6-7. The equations for each zone are:

Zone I

$$q_I = k_I \frac{(h_w - h_1)}{\cot \beta} \ln \left(\frac{h_d}{h_d - h_1} \right) \quad (6-1)$$

30 Sep 86

Zone II

$$q_{II} = \frac{k[h_1^2 - (a_o + h_o)^2]}{2b + 2[h_d - (a_o + h_o)] \cot \alpha} \quad (6-2)$$

Zone III

for $h_o > 0$

$$q_{III} = \frac{ka_o}{\cot \alpha} \left[1 + \ln \left(\frac{a_o + h_o}{a_o} \right) \right] \quad (6-3)$$

for $h_o = 0$

$$q_{III} = \frac{ka_o}{\cot \alpha} \quad (6-4)$$

It is assumed that a , β , b , h_d , h_w , h_o , and k are known for a given problem, thus since $q_I = q_{II} = q_{III} = q$ (continuity of flow, steady state conditions) only a_o , h_1 , and q are unknown. This analysis provides three equations, (6-1), (6-2), and (6-3), or (6-4), and three unknowns. The equations may be solved in a number of ways. One method for $h_o = 0$ is to equate (6-1) and (6-4) and solve for a then equate (6-2) and (6-4) and solve for a_o :

$$\text{from (6-1) and (6-4)} \quad a_o = \frac{\cot \alpha}{\cot \beta} (h_w - h_1) \ln \left(\frac{h_d}{h_d - h_1} \right) \quad (6-5)$$

$$\text{from (6-2) and (6-4)} \quad a_o = \frac{b}{\cot \alpha} + h_d - \sqrt{\left(\frac{b}{\cot \alpha} + h_d \right)^2 - h_1^2} \quad (6-6)$$

then a plot of a_o versus h_1 may be made of equations (6-5) and (6-6). The intersection of the two curves representing (6-5) and (6-6) is the value of a_o and h_1 for solution. Equation (6-4) will then provide q . An example from Harr (1962) is provided in figure 6-8.

30 Sep 86

Example: Determine the quantity of seepage, q , a_o , and h_1 , using Pavlovsky's solution for the dam shown above. Compare with Schaffernak-Van Iterson and L. Casagrande Method, figure 6-2.

A. Pavlovsky's solution

For this case: $\cot \alpha = \cot \beta = 3$ $b = 20'$ $h_d = 80'$ $h_w = 70'$
 $h_o = 0$ $k = 0.002$ ft/min.

For assumed values of h_1 in equations (6-5) and (6-6) the resulting values for a_o are given in the following tabulation and plotted in the accompanying graph.

Equation (6-5)		Equation (6-6)	
h_1	a_o	h_1	a_o
50	19.6	50	15.9
52.5	18.7	52.5	17.7
55	17.4	55	19.7
60	13.9	60	24.1
65	8.4	65	29.3

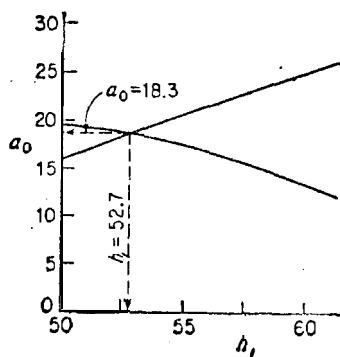
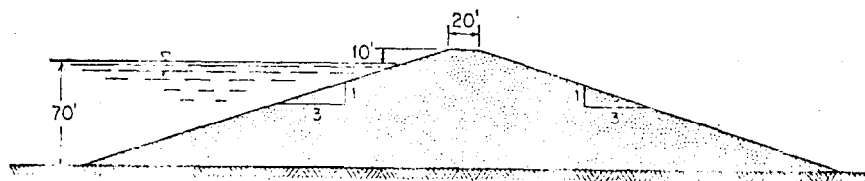


Figure 6-8. Example problem of homogeneous embankment on an impervious foundation comparing solutions (Continued)
 (prepared by WES)

30 Sep 86

From equation (6-4)

$$q = \frac{0.002 \text{ ft/min } 18.3 \text{ ft}}{3} = 0.0122 \text{ ft}^3/\text{min per ft of embankment length}$$

$$a = 3a_o = 54.9 \text{ ft}$$

B. Schaffernak-Van Iterson solution:

$$\text{since } m = 3(70 \text{ ft}) = 210 \text{ ft}$$

$$d = 0.3(210 \text{ ft}) + 3(10 \text{ ft}) + 20 \text{ ft} + 3(80 \text{ ft}) = 353 \text{ ft}$$

$$h_w = 70 \text{ ft}$$

$$a = \frac{353 \text{ ft}}{\cos 18^\circ 26'} - \sqrt{\frac{(353 \text{ ft})^2}{\cos^2 18^\circ 12'} - \frac{(70 \text{ ft})^2}{\sin^2 18^\circ 26'}}$$

$$a = 73 \text{ ft}$$

$$q = (0.002 \text{ ft/min})(73 \text{ ft})(\sin 18^\circ 26')(\tan 18^\circ 26')$$

$$q = 0.015 \text{ ft}^3/\text{min per ft of embankment length}$$

C. L. Casagrande solution:

$$s_o = \sqrt{(353 \text{ ft})^2 + (70 \text{ ft})^2} = 360 \text{ ft}$$

$$a = 360 \text{ ft} - \sqrt{(360 \text{ ft})^2 - \frac{(70 \text{ ft})^2}{\sin^2 18^\circ 26'}}$$

$$a = 76 \text{ ft}$$

$$q = (0.002 \text{ ft/min})(76 \text{ ft} \sin^2 18^\circ 26')$$

$$q = 0.015 \text{ ft}^3/\text{min per ft of embankment length}$$

Figure 6-8 (Concluded)

30 Sep 86

6-3. Earth Dam with Horizontal Drain on Impervious Foundation. Figure 6-5 presents this case for a homogeneous embankment. However, since most earth dams are built in horizontal layers, they very likely have a stratified structure which may allow considerable flow to bypass the horizontal drain, figure 6-9. The difference in vertical and horizontal permeabilities. This can strongly affect the location of the upper line of seepage, figure 6-10, which affects stability considerations and methods of controlling seepage.

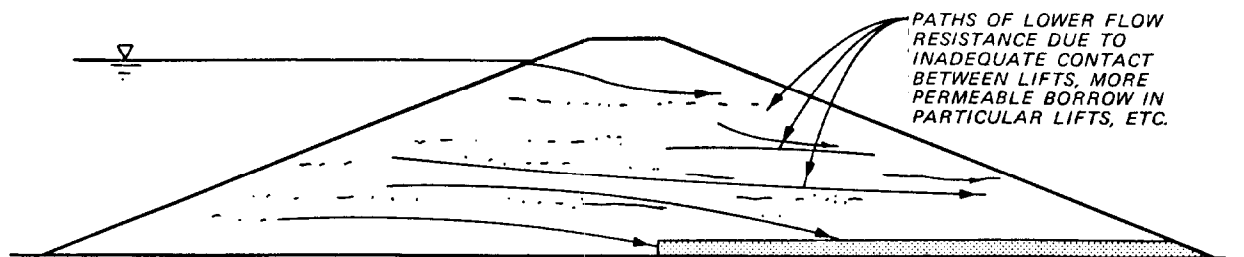


Figure 6-9. Schematic of effect of earth dam stratification on flow of water through embankment with horizontal drain (prepared by WES)

6-4. Earth Dam with Toe Drain on Impervious Foundation. Toe drains are another method of controlling the line of seepage, figure 6-11. Again the effects of anisotropy must be considered, figure 6-12. The geometry of the embankment and the toe drain, height of reservoir, and the degree of anisotropy will control the location of the line of seepage.

6-5. Earth Dam with Vertical or near Vertical Horizontal Drains on Impervious Foundation. One very effective method of intercepting horizontal flows due to stratification of the embankment, figure 6-9, is the incorporation of an inclined or vertical drain into the central portion of the embankment, figure 6-13. This seepage analysis of a zoned, anisotropic embankment assumed the rockfill to have infinite permeability with respect to the core materials and used the method recommended by A. Casagrande for drawing the parabola to determine the upper line of seepage. The interface of the core and inclined drain is used as the downstream slope for the seepage face since the drain has a much higher permeability than the core material. Provision must be made in sizing the drain to pass all the water coming out of the core without building up a tailwater on the downstream slope of the core. It can be noted that the designers of this example used $m/3$ instead of 0.3m to determine the interception point of the parabola and headwater elevation and the formula

$$a + \Delta a = \frac{y_o}{1 - \cos \alpha}$$
 which can be derived trigonometrically, to determine $a + \Delta a$. The geometry of the embankment and toe drain, height of reservoir, and the degree of anisotropy will control the location of the line of seepage.

6-6. Flow Net for a Composite Zoned Dam. If differences in permeabilities between zones in a zoned dam are great enough (e.g. 100 to 1000 times or more) the more permeable zone may be considered to have infinite permeability relative to the less permeable zone for purposes of seepage analysis. In the seepage analysis example of figure 6-13 the upstream rockfill and the inclined

30 Sep 86

Note: Flow net is for isotropic case ($k_h = k_v$)

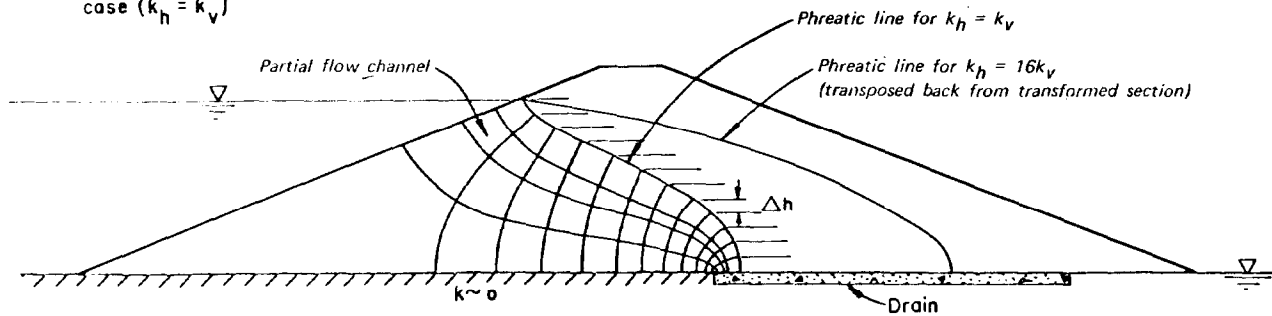


Figure 6-10. Effect of difference in horizontal and vertical permeability (anisotropy) on location of line of seepage within an embankment with a horizontal drain (after U. S. Department of Agriculture¹²³)

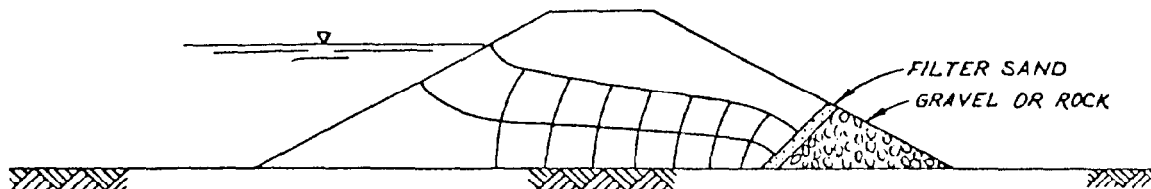


Figure 6-11. Homogeneous embankment on impervious foundation with a toe drain (from EM 1110-2-1913¹¹)

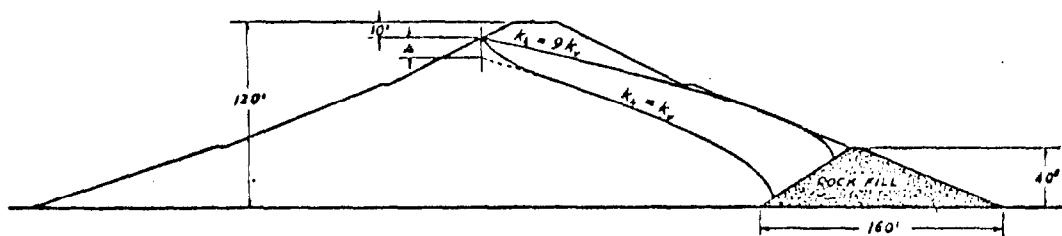


Figure 6-12. Effect of anisotropy for homogeneous embankment on impervious foundation with a toe drain (courtesy of New England Waterworks Association¹⁵¹)

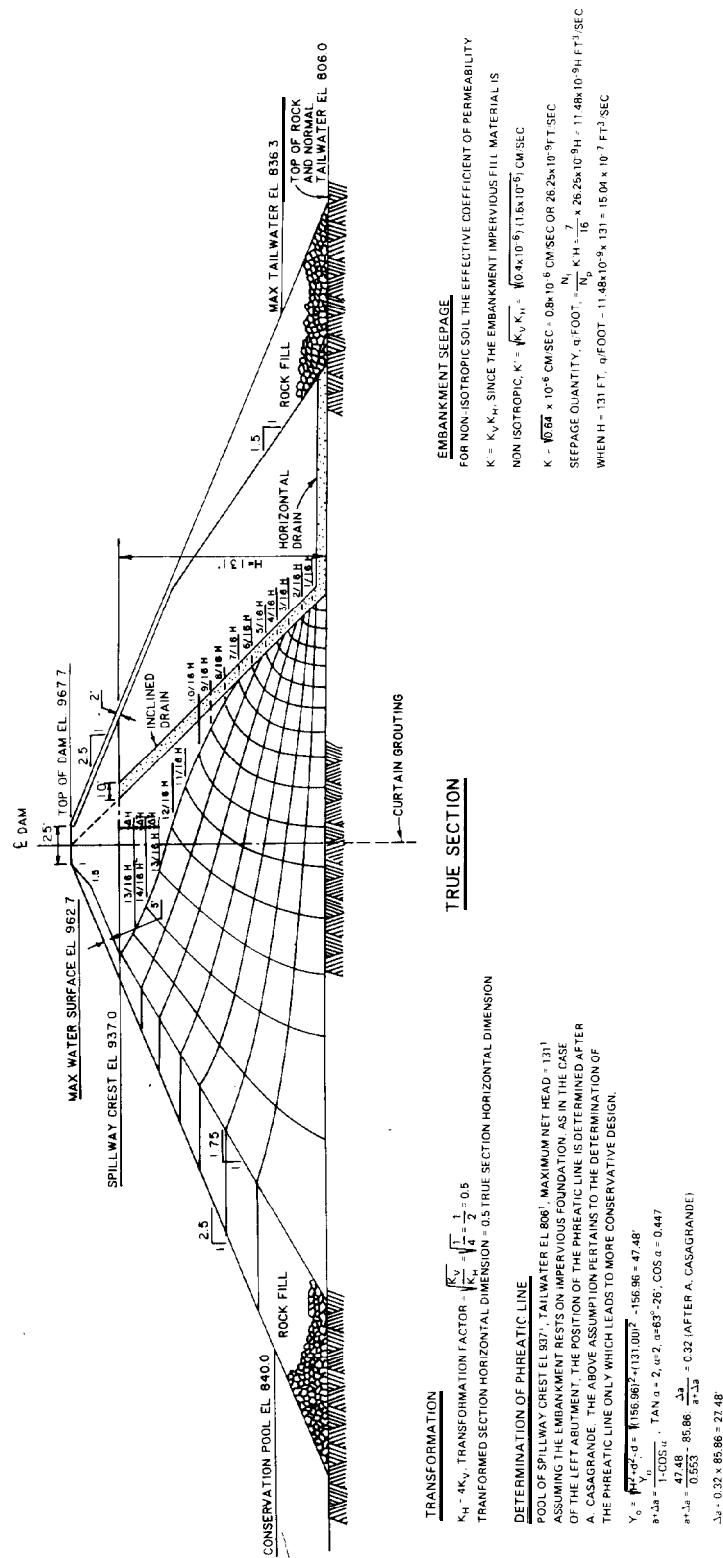


Figure 6-13. Seepage analysis for Kettle Creek earth embankment, Kettle Creek, Pa. (from U. S. Army Engineer District, Baltimore 78)



FLOW NET

LEFT ABUTMENT-STATION 17+00

TRANSFORMATION

$$K_H = 4K_V, \text{ TRANSFORMATION FACTOR} = \sqrt{\frac{K_V}{K_H}} = \sqrt{\frac{1}{4}} = \frac{1}{2} = 0.5$$

TRANSFORMED SECTION HORIZONTAL DIMENSION = 0.5 TRUE SECTION HORIZONTAL DIMENSION

DETERMINATION OF PHREATIC LINE

POOL OF SPILLWAY CREST EL 937¹, TAILWATER EL 806¹, MAXIMUM NET HEAD = 131¹ ASSUMING THE EMBANKMENT RESTS ON IMPERVIOUS FOUNDATION, AS IN THE CASE OF THE LEFT ABUTMENT, THE POSITION OF THE PHREATIC LINE IS DETERMINED AFTER A CASAGRANDE. THE ABOVE ASSUMPTION PERTAINS TO THE DETERMINATION OF THE PHREATIC LINE ONLY WHICH LEADS TO MORE CONSERVATIVE DESIGN.

$$Y = \sqrt{H^2 + d^2} - d = \sqrt{(156.96)^2 + (131.00)^2} - 156.96 = 47.48'$$

$$a + \Delta a = \frac{Y_o}{\cos \alpha}, \quad \tan \alpha = 2, \alpha = 63^\circ - 26', \cos \alpha = 0.447$$

$$a + \Delta a = \frac{47.48}{0.553} = 85.86; \quad \frac{\Delta a}{a + \Delta a} = 0.32 \text{ (AFTER A. CASAGRANDE)}$$

$$\Delta a = 0.32 \times 85.86 = 27.48'$$

EMBANKMENT SEEPAGE

FOR NON-ISOTROPIC SOIL THE EFFECTIVE COEFFICIENT OF PERMEABILITY

$K' = K \cdot K_{\infty}$. SINCE THE EMBANKMENT IMPERVIOUS FILL MATERIAL IS

NON ISOTROPIC. $K' = \sqrt{K_V K_H} = \sqrt{(0.4 \times 10^{-6}) (1.6 \times 10^{-6})}$ CM/SEC

$$K' = \sqrt{64} \times 10^{-6} \text{ CM/SEC} = 0.8 \times 10^{-6} \text{ CM/SEC OR } 26.25 \times 10^{-9} \text{ FT/SEC}$$

$$K'H = \frac{N_f}{\alpha} \times 26.25 \times 10^{-9} H = 11.48 \times 10^{-9} H \text{ FT}^3/\text{SEC}$$

SEEPAGE QUANTITY, $q/\text{FOOT} = \frac{K \cdot H}{N_p} = \frac{16}{16}$

WHEN $H = 131 \text{ FT}$ $q/\text{FOOT} = 11.48 \times 10^{-9} \times 131 = 15.04 \times 10^{-7} \text{ FT}^3/\text{SEC}$

Figure 6-13. Concluded

30 Sep 86

drain are considered infinitely permeable with respect to the central earth portion of the embankment. In some cases dams may contain adjoining zones which, have relatively small but marked differences in permeability and it may be desired to accurately analyze the flow through these zones. Equipotential lines and flow lines, with the exception of the upper line of seepage, will cross the interface of the zones in the same manner as given for confined composite sections in Chapter 4, but the location of the upper line of seepage (phreatic line) must first be determined. Once its location is determined the upper line of seepage transfers between regions of different permeabilities in the manner shown in figure 6-14. In determining the location of the upper seepage line, the essential principle to remember is that the flow rate must be the same through each zone. That is, for a unit depth of embankment perpendicular to the plane of the flow net, Q must be the same for each zone. More permeable zones require less gradient and/or cross-sectional area to pass the flow transmitted to them from less permeable zones. This idea can be seen in the example and instructions taken from Cedergren 1977.

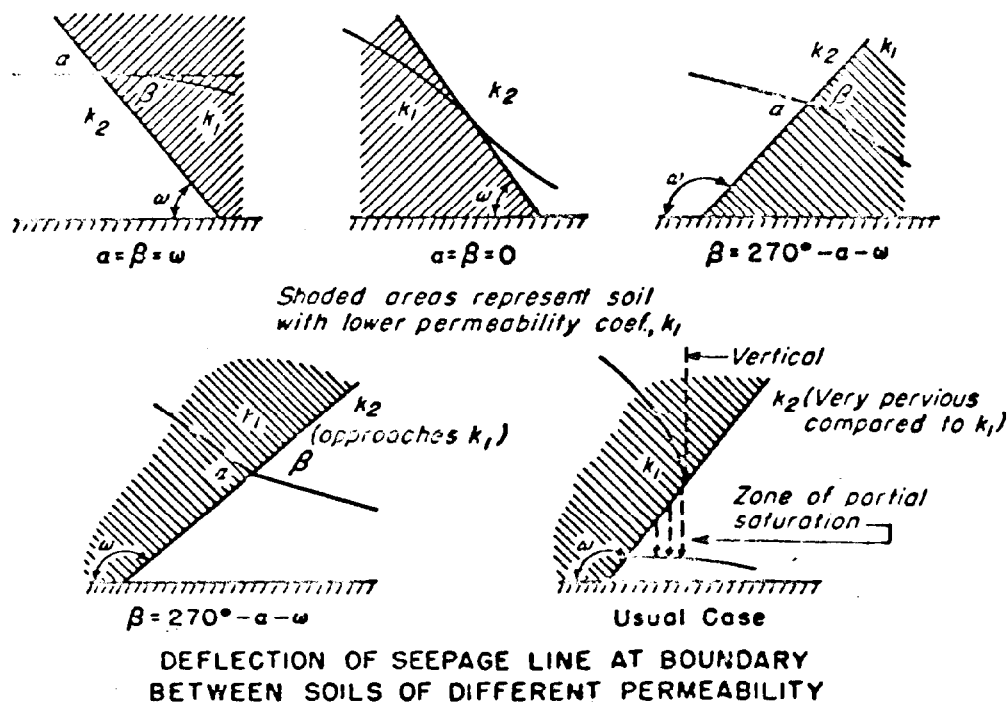


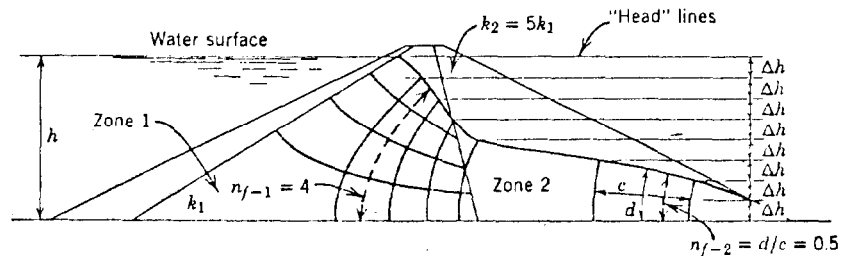
Figure 6-14. Transfer of line of seepage between regions of different permeability (courtesy of New England Waterworks

Association¹⁵¹)

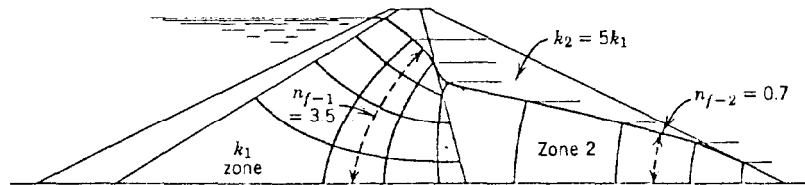
a. Locate the reservoir level and the tailwater level, noting the difference in head as h and dividing h into a convenient number of equal parts of increments Δh . Draw a series of light horizontal guide lines (head lines) at intervals of Δh across the downstream part of the section (figure 6-15a).

30 Sep 86

b. Guess a trial position for the phreatic line in both zones and draw a preliminary flow net as shown in figure 6-15a, making squares in zone 1 and rectangles in zone 2. Make the length-to-width ratios of all of the rectangles in zone 2 approximately equal by adjusting the shape of the saturation line, using an engineer's scale to measure the lengths and widths of the figures. When this step is completed, the trial flow net should be reasonably well drawn. It should satisfy the basic shape requirements of a flow net, but the length-to-width ratio of the shapes in zone 2 probably will not satisfy $c/d = k_2/k_1$. Although the flow net has been drawn for a composite section, the ratio of k_2/k_1 probably does not equal the k_2/k_1 ratio originally assumed for the section.



a. First trial-flow net (not correct)



b. Completed flow net (correct)

Figure 6-15. Method for constructing flow nets for composite sections (courtesy of John Wiley and Sons¹⁵⁵)

c. Calculate the actual ratio of k_2/k_1 for the trial flow net just constructed. To make this important check proceed as follows:

(1) Count the number of full flow channels between any two adjacent equipotential lines in zone 1 and call this number n_{f-1} . In the trial flow net in figure 6-15a, $n_{f-1} = 4.0$.

(2) Count the number of full flow channels between any two adjacent equipotential lines in zone 2 and call this number n_{f-2} . In figure 6-15a, n_{f-2} is equal to the width-to-length ratio of the figures in zone 2, d/c , and equal to 0.5.

(3) The actual value of k_2/k_1 for the trial flow net in figure 6-15a can now be determined from the equation

30 Sep 86

$$k_2 = k_1 \frac{n_{f-1}}{n_{f-2}}$$

or

$$\frac{k_2}{k_1} = \frac{n_{f-1}}{n_{f-2}}$$

(4) If the calculated k_2/k_1 ratio is too high, the saturation line in zone 2 is too low and must be raised. If the calculated k_2/k_1 ratio is too low, the saturation line in zone 2 is too high and must be lowered. Raise or lower the general level of the saturation line in zone 2 as indicated and construct another trial flow net.

(5) Repeat steps (1) through (4) until a flow net of the desired accuracy is obtained. (Usually a few trials will be sufficient.) By applying the above equation to the first trial flow net in this example (figure 6-15a) $k_2/k_1 = 4.0/0.5 = 8.0$. Because the ratio of k_2/k_1 for this example was assumed to be 5, the k_2/k_1 ratio of the trial flow net is too high; hence the general level of the saturation line in zone 2 is too low and must be raised. For the second trial flow net (figure 6-15b) $n_{f-1} = 3.5$ and $n_{f-2} = 0.7$. The calculated ratio of $k_2/k_1 = 3.5/0.7 = 5.0$, the value originally assumed. The above equation may be derived by recalling that the quantity of seepage in zones 1 and 2 (figure 6-15) must be equal. Using

$$q = kh \frac{n_f}{n_d}$$

in zone 1, $q = k_1 h (n_{f-1}/n_d)$, and in zone 2, $q = k_2 h (n_{f-2}/n_d)$. For a given head h , $q \sim k_1 (n_{f-1}/n_d) \sim k_2 (n_{f-2}/n_d)$ and $q/n_d \sim k_1 n_{f-1} \sim k_2 n_{f-2}$. Therefore $k_1 n_{f-1} = k_2 n_{f-2}$ and $k_2 = k_1 (n_{f-1}/n_{f-2})$. This expression can be used for determining the permeability ratios k_2/k_1 , k_3/k_2 , and so on, for any composite flow net being examined for accuracy. It is an essential criterion to be used in constructing accurate flow nets for composite sections. Figure 6-16 illustrates the additional detail which can be obtained by further subdivision of the flow net. This may be necessary for a stability analysis or other reasons. Another example provided in figures 6-17 and 6-18 illustrates a flow net for a zoned dam on an impervious foundation; figure 6-17 is the transformed section for the true section of figure 6-18 (Cedergren 1975 and U. S. Army Engineer District, Sacramento 1977).

30 Sep 86

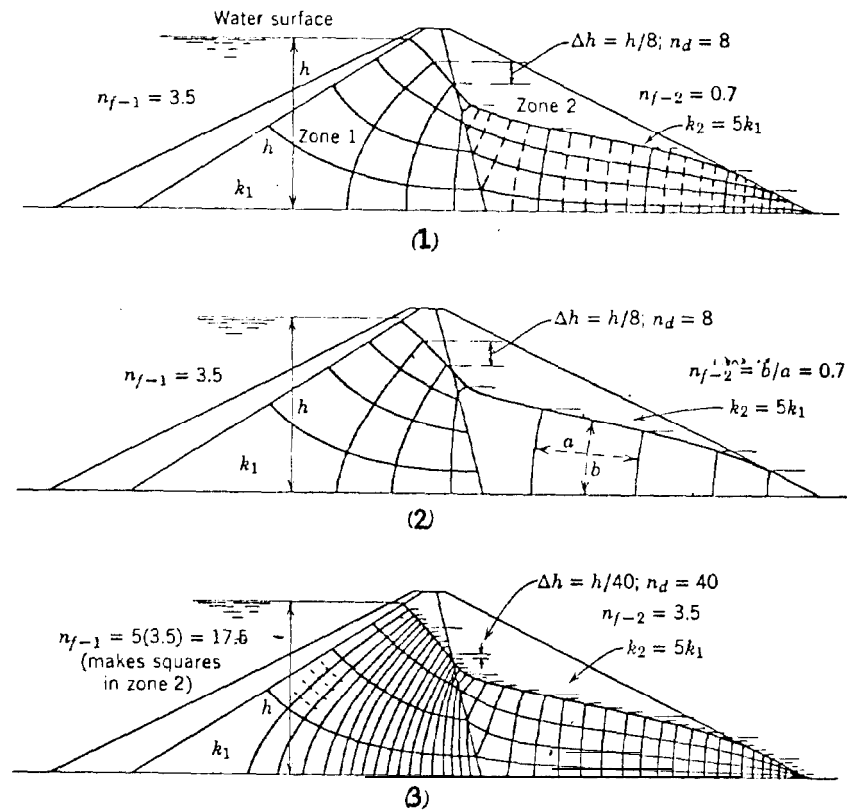


Figure 6-16. Three forms of one flow net (courtesy of John Wiley and Sons¹⁵⁵)

6-7. Zoned Earth Dam on Pervious Foundation.

a. Zoned embankments on pervious foundations may require rather involved seepage analyses unless simplifying assumptions are made. This is particularly true when relatively small permeability differences exist between adjacent zones and foundation and it is desired to consider these differences in permeability. Figure 6-19 provides an example of such an analysis (U. S. Army Engineer Waterways Experiment Station 1956b). Development of the flow net required use of a number of principles and methods explained in Chapter 4, and previous portions of Chapter 6, e.g., dimensional changes due to anisotropy, upper seepage lines and other flow lines crossing the interface of materials having different permeabilities and composite sections. It should be noted that several assumptions were made.

(1) The flow net for the foundation was drawn independently of the embankment, i.e. considering the embankment to be impermeable, and assuming the foundation to be isotropic.

(2) The embankment flow net reflects the influence of foundation flow net in the location of equipotential and flow lines and is drawn for anisotropic conditions.

30 Sep 86

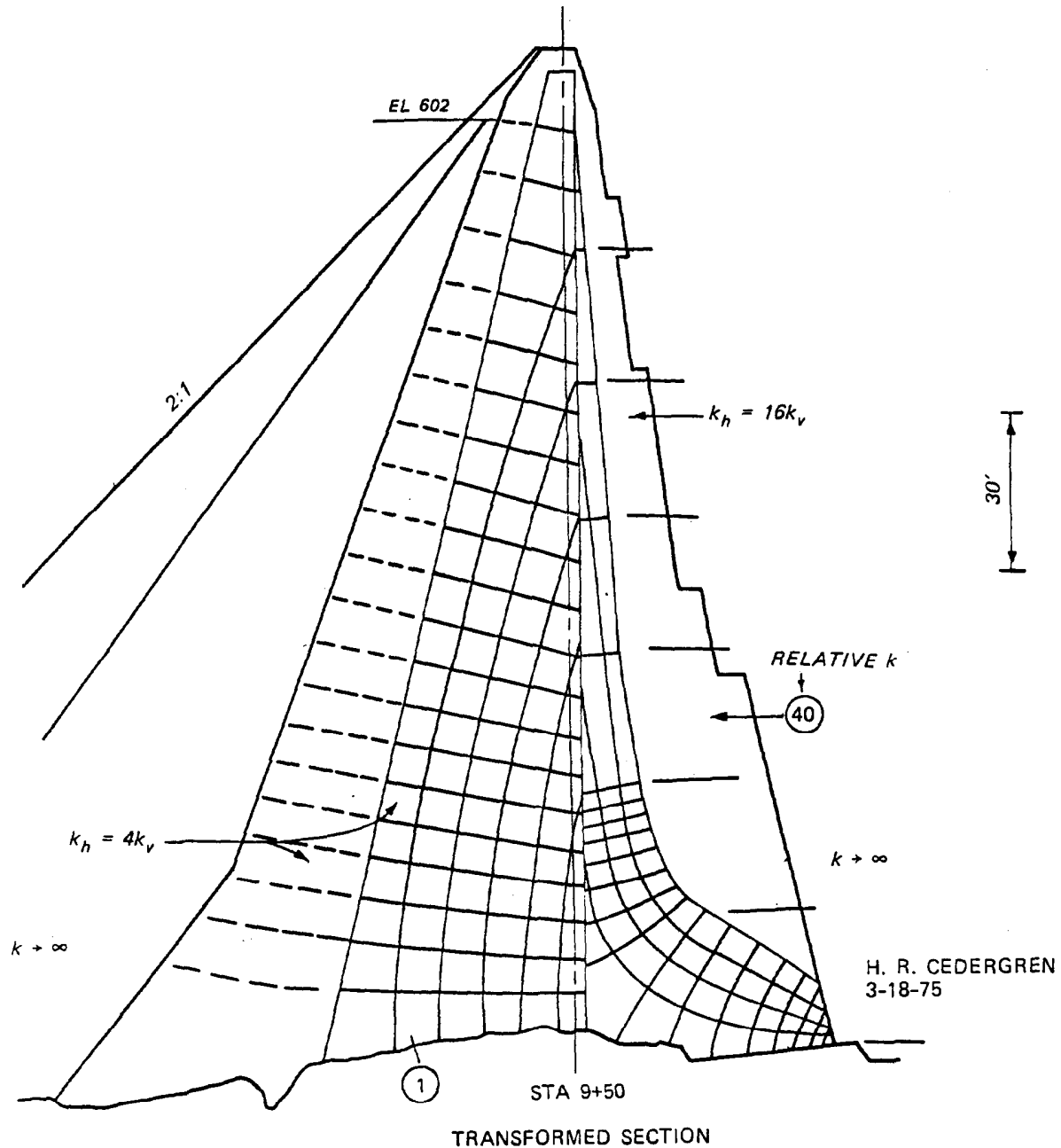
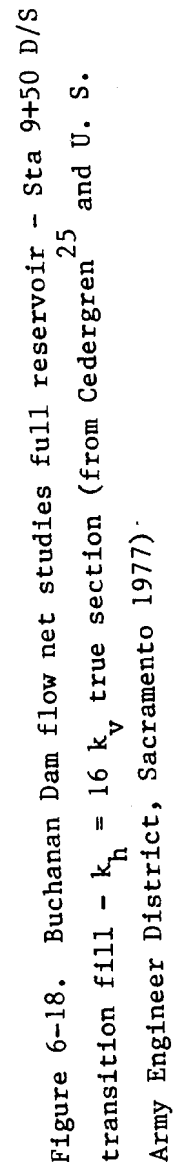


Figure 6-17. Buchanan Dam flow net studies full reservoir- Sta 9+50 D/S transition fill - $k_h = 16 k_v$ transformed section (from Cedergren²⁵ and U. S. Army Engineer District, Sacramento 1977)



30 Sep 86

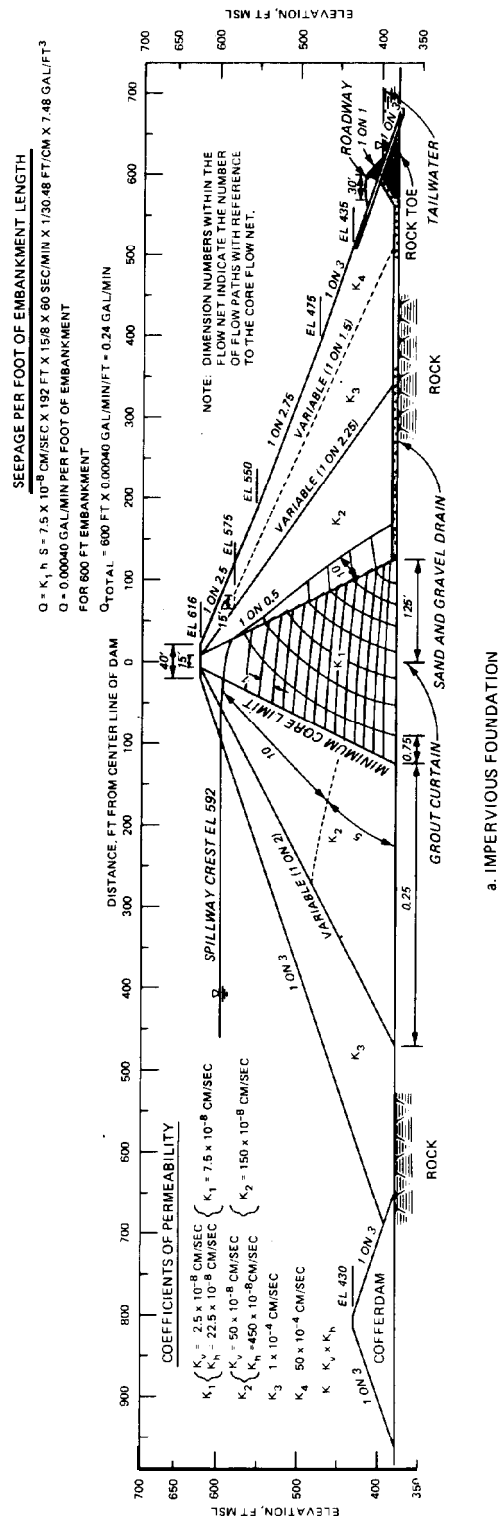


Figure 6-19. Seepage analysis, Blakely Mountain Dam, Arkansas (from U. S. Army Engineer Waterways Experiment Station ¹²¹)

30 Sep 86

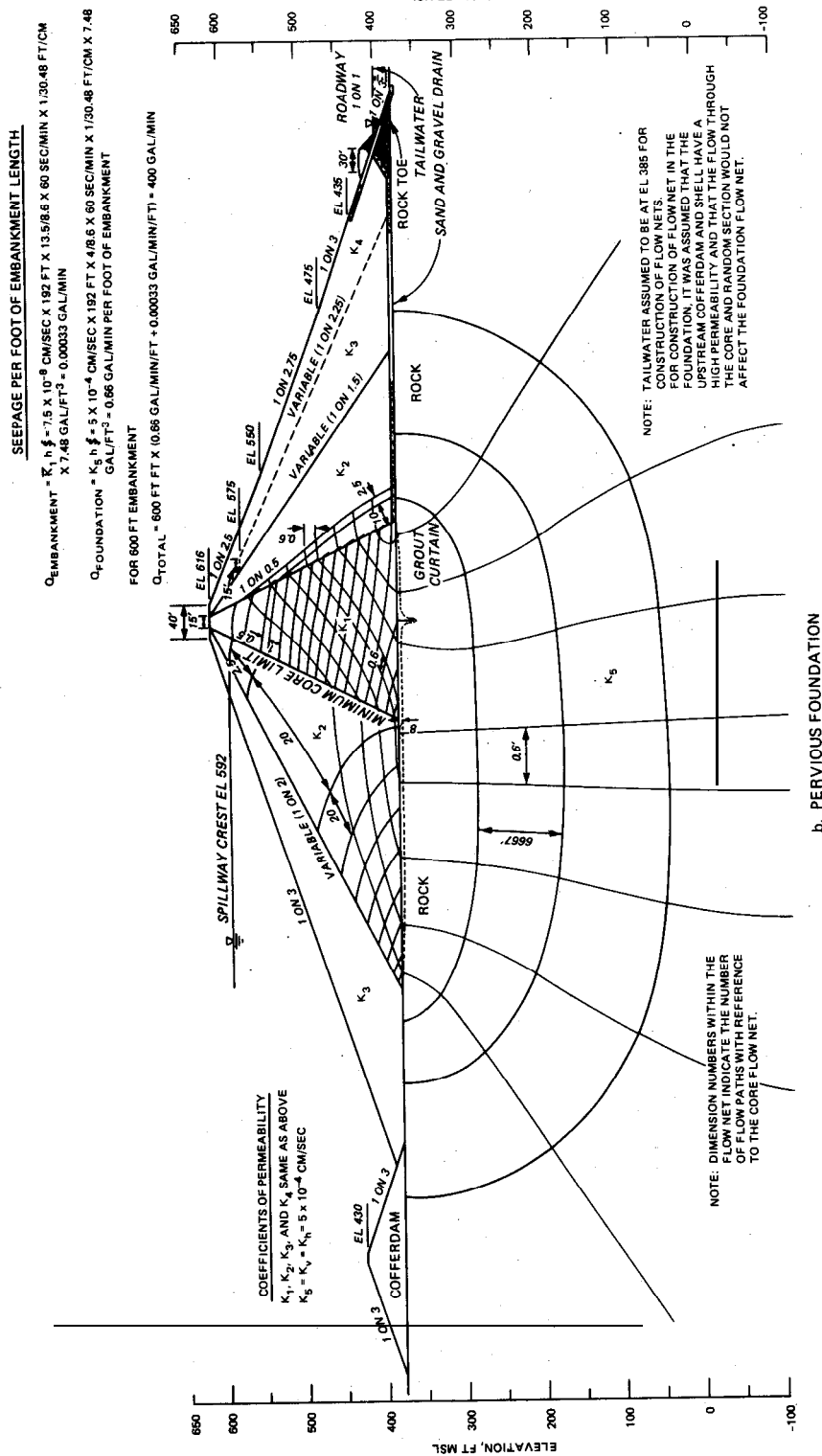


Figure 6-19. Concluded

30 Sep 86

(3) The flow nets were constructed assuming a tailwater elevation of 388 ft (approximate elevation of the base of the embankment) while seepage quantities calculations assumed a tailwater elevation of 400 ft as shown on the cross sections (i.e., h for calculation of Q is 192 ft).

(4) For calculation of Q in figure 6-19(b) the flow nets were considered to be separate but to have the same number of equipotential drops.

b. This example provides evidence that a large permeable member of the embankment-foundation material controls the quantity of flow. In this case the relatively impermeable embankment materials needed to be considered in order to determine the upper seepage line, the extent of saturated materials, and the expected pore pressures within the embankment. Seepage for this problem might also be evaluated using a finite element computer program.